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SCE&G®
A SCANA COMPANY

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U.S. Nuclear Regulatory Commission
Document Control Desk
Washington, DC 20555

ATTN: Document Control Desk

Subject: Virgil C. Summer Nuclear Station (VCSNS) Units 2 and 3 Combined License Application (COLA) - Docket Numbers 52-027 and 52-028 Response to NRC Request for Additional Information (RAI) Letter No. 056

Reference: Letter from Chandu P. Patel (NRC) to Alfred M. Paglia (SCE&G), Request for Additional Information Letter No. 056 Related to SRP Section 2.5.4 for the Virgil C. Summer Nuclear Station Units 2 and 3 Combined License Application, dated July 9, 2009.

The enclosure to this letter provides the South Carolina Electric & Gas Company (SCE&G) response to the RAI items included in the above referenced letter. The enclosure also identifies any associated changes that will be incorporated in a future revision of the VCSNS Units 2 and 3 COLA.

Should you have any questions, please contact Mr. Al Paglia by telephone at (803) 345-4191, or by email at apaglia@scana.com.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on this 7th day of August, 2009.

Sincerely,

Ronald B. Clary

Ronald B. Clary
General Manager
New Nuclear Deployment

JMG/RBC/jg

Enclosure

D083
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NRC RAI Letter No. 056 Dated July 9, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

NRC RAI Number: 02.05.04-34

In response to RAI 02.05.04-2, you stated that saprolite soil column was used in site seismic response analysis to obtain peak ground acceleration and the generic EPRI soil degradation curves used in the SHAKE analysis agree well with the results of the three RCTS tests performed on saprolite soil samples. However, some RCTS test results show that the EPRI curves used in SHAKE analysis for saprolie soil give higher shear modulus and higher damping ratio for shear strain greater than 0.003, which indicates that the EPRI soil degradation curves used in site seismic response analysis are not conservative. Please verify the impact of using saprolite soil degradation property based on RCTS results instead of EPRI generic curves on SHAKE analysis results.

VCSNS RESPONSE:

As discussed in the RAI 02.05.04-2 response, 69% of the saprolite consists of silty sand and the two RCTS results on the silty sand samples agree well with the EPRI curves (for both G/Gmax and D) used in the SHAKE analysis. The response also indicated that the RCTS test results for the third saprolite sample tested, namely the elastic silt sample (B-208-UD3), agree better with the generic EPRI clay curves (the curves for plasticity index PI = 50 in this case) than the generic EPRI sand curve. The response also explains that elastic silt is one of several materials (including low plasticity silts and some clays) that make up the remaining 31% of the saprolite, and the RCTS test on the elastic silt was made for completeness. Of the 74 saprolite samples tested for plasticity, 62 were classified as non plastic - these were mainly silty sands. Of the remaining 12 samples, the B-208-UD3 sample had the highest plasticity index, i.e., this was the most plastic of the 72 samples tested. Note that the 31% non silty sand materials do not occur in a discrete layer since they are the result of a weathering in place process.

The SHAKE analysis of the 25 to 35 feet of saprolite was run with the properties of the silty sand, i.e., with properties reflecting the large majority of the saprolite. Running the analysis using the properties of the elastic silt would give a distorted result. Note also that the G/Gmax and D versus shear strain curves used in the SHAKE analysis were randomized, i.e., 60 different curves varying about the median were used. The G/Gmax curve from the RCTS test results for the B-208-UD3 sample falls outside the set of randomized curves; however, the damping ratio curve from the RCTS test results falls very close to the limiting randomized damping ratio curve used in the SHAKE analysis.

As explained in the RAI 02.05.04-2 response and in the FSAR, all of the saprolite is being removed from beneath and around the Nuclear Island, and from beneath and around all of the major power block structures. The seismic response of the saprolite is not used in the analysis of these structures.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA revisions have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 056 Dated July 9, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

NRC RAI Number: 02.05.04-35

During the site audit of March 31 to April 1, 2009, we noted that (1) the settlement calculations were based on AP1000 DCD Revision 15 design parameters but the current revision 17 updated the parameters, such as maximum static foundation pressure and settlement design parameters; (2) the settlement was calculated using assumed rectangular shapes for nuclear island foundation although the foundation shape is irregular; (3) the settlement calculations were only performed at two points for each foundation – one at the center and one along one side with the shortest distance from the center; (4) the applicant mentioned during the site audit that high-strain modulus was used in the foundation settlement calculations, but the FSAR did not specify this; and (5) the turbine building is founded on deep soil at one end and hard rock on the other.

Since foundation settlement is an important parameter in structure and foundation stability and safety evaluation, please provide the following information: (1) re-evaluate foundation settlement in accordance with the parameters defined in AP1000 DCD Revision 17; (2) evaluate the impact of using actual nuclear island foundation shape, compared with using rectangular shape, in settlement calculation; (3) verify whether 2-location settlement calculation for a foundation can fully describe whole foundation total and differential settlements; (4) clarify the shear strain level at which the soil moduli under the foundation were determined, and describe how this shear strain level compared with the stain level corresponding to the site GMRS seismic loading; and (5) evaluate the potential impact of the nonuniform support condition on settlement of the turbine building.

VCSNS RESPONSE:

(1) The only change in the Revision 17 bearing parameters is the static bearing demand over the footprint of the Nuclear Island, which has increased from 8.6 to 8.9 ksf, i.e., a 3.5% increase. The average computed total settlement based on 8.6 ksf was 0.013 in. This was rounded up to 0.015 in. in the FSAR. The average computed total settlement for 8.9 ksf demand is 0.014 in. Thus, the FSAR value of 0.015 in. would not change for the increased loading, but the increased loading will be noted in the FSAR. Note that 0.015 in. is <1/50 in. Revision 17 of the AP1000 DCD allows a total settlement of 3 inches for the Nuclear Island.

(2) The settlement was computed using the maximum width x length of the Nuclear Island footprint, thus enveloping the settlement using the actual Nuclear Island foundation shape. As noted in response (1) to this RAI, the average computed total settlement for 8.9 ksf demand is 0.014 in. and the AP1000 DCD allowable settlement is 3 in. The impact of using the actual Nuclear Island foundation shape in the settlement calculation would be to reduce the average computed total settlement to less than 0.014 in.

(3) Corner settlement estimates are sometimes also included in addition to estimated center and edge settlement values. The corner settlements were not included in the FSAR since the corner settlement values are always less than the center and edge values for a uniformly loaded structure. Including the corner settlement would slightly lower the average computed settlement for each structure and would slightly decrease the computed differential settlement between the Nuclear Island and the other buildings. It would slightly increase the computed maximum differential settlement across the Nuclear Island; however, since the computed maximum differential settlement is about 0.004 in. per 50 ft. using the corner settlement, and the allowable differential settlement from Revision 17 of the AP1000 DCD is 0.5 in. per 50 ft., this would have no impact.

(4) For sound bedrock, the elastic modulus is essentially independent of strain. Thus, in the settlement analyses of the various structures, only the computed settlement of structures underlain by structural fill will be affected by the strain level in the fill. The conservative settlement values obtained using the high strain modulus given in the FSAR give settlement values for the structures on structural fill that are within tolerable limits. (The FSAR will be revised as indicated below to specifically state that these are high strain values.) Thus, there is no need to use the strain dependent modulus analysis which would give smaller settlements. These are static analyses and so the seismic strain levels are not relevant.

(5) The non-uniform support condition on settlement of the Turbine Building was discussed in the response to RAI 02.05.04-28. The fact that there is rock at or close to the bottom of one portion of the building is relevant to the differential settlement of the building only to the extent that it minimizes this differential settlement since there is essentially no settlement in the underlying rock. In other words, the differential settlement is caused solely by the difference in thickness of structural fill beneath the foundation and differences in loading on various parts of the structure.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

1. The last sentence of the 1st paragraph of FSAR Subsection 2.5.4.10.1.3 has been revised in COLA Revision 1 to read as follows:

   "For the Nuclear Island, the value on Table 2.5.4-219 exceeds the required allowable static and dynamic bearing capacities ~~of 8.6 ksf and 35 ksf, respectively~~, given in Table 2-1 of the AP1000 DCD."

2. The 4th paragraph, next to last sentence of FSAR Subsection 2.5.4.10.2 has been revised in COLA Revision 1 to read as follows:

   "The applied pressure used in the settlement computation for the Nuclear Island foundation is ~~8.6 ksf,~~ from Table 2-1 of the AP1000 DCD."

3. The column labeled "Contact Pressure (ksf)" for the "Nuclear Island" row in FSAR Table 2.5.4-220 has been revised in COLA Revision 1 from "8.6" to "8.9".

4. The 2nd paragraph of FSAR Subsection 2.5.4.10.2 will be revised in a future COLA update to read:

   E_i = elastic modulus of layer i <u>(high strain value used)</u>

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 056 Dated July 9, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

NRC RAI Number: 02.05.04-36

We noted that (1) In your response to RAI 02.05.04-3, you stated that some portions of sound (nonrippable) rock left above foundation base level after excavation "will be removed down to foundation base level using controlled blasting." But in response to RAI 02.05.04-14, you stated that "[h]owever, this [controlled blasting] would not be practical since the thickness of rock to be removed would not be known. In addition, even controlled blasting would have some impact, such as minor fracturing, on the immediately underlying rock, which would be less desirable than letting the thin transition zone remain in place." Those two statements are inconsistent. (2) In your response to RAI 02.05.04-3, you stated that "where needed, the concrete placed between the top of sound rock and the base of the nuclear island will have a strength of about 5,000 psi. One reason for selecting this strength is that, according to Boone (2005), concrete with this strength has a shear wave velocity of around 9,000 fps [2,743 m/s], i.e., close to that of the in-situ rock." Based on our confirmatory calculations, in accordance with ACI 318-08, for unreinforced concrete with strength of 34,483 kPa (5,000 psi), the corresponding shear wave velocity is about 2,225 m/s (7,300 fps) – far less than that estimated by you. (3) In your response to RAI 02.05.04-4, you indicated that the concrete fill will have maximum thickness of 1.52 to 5.18 m (5 to 17 ft) underneath the nuclear island. For unreinforced concrete fill with such thickness, cracking will be a serious distress to be concerned about.

In order for us to fully evaluate the adequacy of the foundation, please explain (a) how the foundation base level will be reached to meet the "sound rock" criterion; (b) how you ensure that the proposed concrete fill will have properties to sound rock to meet the uniformity requirement; and (c) how the proposed concrete fill will be designed to reduce cracking distress and to ensure long term strength and stability.

VCSNS RESPONSE:

(1) If above foundation level the rock is non-rippable, then by definition the only way to reach foundation level is by controlled blasting, with every precaution being taken not to impact the underlying rock. If below foundation level the rock is non-rippable, then there is a choice of leaving this in-situ rock in place or removing it by blasting and replacing it with concrete. The response to RAI 02.05.04-14 argues that it is preferable to leave the thin layer of non-rippable rock below the foundation level in place rather than blast it out and replace it with concrete, particularly since the exact thickness of the rock is not known. Whether the in-situ rock is kept in place or replaced with concrete, it will consist of a few feet of

high strength material (10,000 psi rock or 5,000 psi concrete) that will have a shear wave velocity at or relatively close to that of sound rock (9,200 ft/sec). This thin layer would not impact the response analysis to any significant extent.

(2) The relationship between the strength of concrete and its shear wave velocity is empirical. The Boone (2005) equation is derived from actual measurements on concrete specimens. The relationship in ACI 318-08 is between strength and elastic modulus, and the elastic modulus can then be converted to shear wave velocity. Both equations demonstrate that the shear wave velocity for a certain strength of concrete is typically higher than the shear wave velocity of bedrock with the same strength. Since concrete itself consists of different proportions of different coarse and fine aggregates, cement and water (and sometimes flyash as noted in (3)), then it is to be expected that there will be different empirical relationships between strength and shear wave velocity in the literature. Since concrete strength continues to increase with age, the shear wave velocity will also increase with age. Thus, whether a shear wave velocity of 8,990 ft/sec (Boone), 7,300 ft/sec (ACI) or 8,145 ft/sec (average) for 5,000 psi concrete is used, this value will increase during the lifetime of the plant.

The maximum thickness of the concrete will be below Unit 2. Boring logs indicate this maximum thickness will be about 17 ft. Based on the response to RAI 02.05.02-18, this concrete would not significantly impact the seismic response. The response to RAI 02.05.02-18 looked primarily at the impact on the seismic response of a limited thickness of fractured rock beneath the Nuclear Island. For Unit 2, the upper 25 ft of rock ranged in shear wave velocity from about 6,500 ft/sec to 10,500 ft/sec, with an average of about 8,800 ft/sec. Since the concrete has a shear wave velocity range similar to the weathered rock, but is thinner, the effect on the seismic response will be less. The response to RAI 02.05.02-18 showed that for the Unit 2 case, there was zero amplification up to 10 Hz; the amplification then increased to a maximum of about 5% at 50 Hz, and fell to about 3% at 100 Hz. The response concluded that the overall amplification is very small and its impact on the GMRS is negligible. The same would be true for the concrete.

(3) To establish the foundation bearing level for the Nuclear Island (NI), fill concrete will be used beneath the footprint of the NI basemat, and extending a few feet outward. The excavation around the NI and beneath other major power block structures will be backfilled with compacted granular structural fill. The relative concrete and structural fill locations are shown on revised FSAR Figures 2.5.4-220 through 2.5.4-223. The NI fill concrete will extend a several feet (5 or 6 feet), beyond the footprint of the Nuclear Island. Concrete fill will be used between the bottom of the Nuclear Island foundation and the finish grade on sound rock. Based on the top of Layer V (Sound Rock) contours (FSAR Figure 2.5.4-202), the top of sound rock occurs generally at El. 360 +/- 5 ft beneath the NI at Units 2 and 3, but it is approximately 17 ft lower (EL 343 ft) at the northeast corner of

the Unit 2 Nuclear Island and approximately 12 ft higher (EL 372 ft) beneath the southern part of the Unit 2 NI. The NI areas will be excavated in sound rock to approximately EL 357 ft, where required, to allow a minimum 3 ft thickness of fill concrete and mud mat beneath the NI basemats. The fill concrete will be approximately 17 ft thick beneath the northeastern corner of the Unit 2 basemat.

American Concrete Institute (ACI) defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." The definition is intentionally vague because many factors, including the concrete mix design, the dimensions, the type of the placement, and the curing methods, affect whether or not cracking will occur. ACI 207.IR-96, "Mass Concrete," prepared by ACI Committee 207, governs the design and construction of mass concrete. Typically, there are two common concerns associated with thermal cracks in mass concrete. They are: (1) the maximum temperature inside a concrete pour and (2) the maximum temperature difference between the hottest spot and the surface of a concrete pour. Specifications of mass concrete typically limit the maximum temperature to 155^0F and the maximum temperature difference between the interior and the surface to 36^0F, so that early-age thermal cracks in mass concrete will be minimized. It is a common practice to limit the least dimension of each concrete pour so that the temperature and temperature difference of the pour can stay within their respective limits.

According to the ACI mass concrete definition, the fill concrete under the Nuclear Island of V. C. Summer Unit 2 is a mass concrete. A thermal control plan considering the geometry of Unit 2 fill concrete, the proposed 5,000 psi strength, total volume of fill concrete placement, and rate of concrete production, will be prepared to make sure that the rule-of-thumb temperature limits will not be exceeded. The thermal control plan will have the following elements:

- Use well-graded aggregate and Type I and/or II cement in the concrete mix.
- Because of its relatively high strength specification, the fill concrete will likely have a high content of Portland cement substitutes, such as Class F flyash and/or slag, to minimize the heat of hydration.
- In anticipation of variations in elevation in sound rock surface, the minimum thickness of fill concrete will be set at 3 feet, which includes the 6-inch layer of mud mat.
- Even with the heat of hydration in the design mix minimized, it may still require the concrete to be placed in relatively thin lifts to avoid cracking.

Thus, the maximum thickness of each concrete lift will be set at about 5 feet.

- Concrete will be placed using a step technique to minimize the live face of concrete, thus minimizing the chance for cold joints.
- Exposed surfaces of each concrete lift will be insulated, if required.
- When another lift is required on top of an existing lift, the top lift will be poured only after the bottom lift has enough time to properly cool down.
- Concrete placing temperature will be controlled as necessary by use of ice, chilled water, shading aggregate piles, spraying coarse aggregate for evaporative cooling, and scheduling placements (such as at night) to take advantage of coolest temperatures.
- Planned vertical joints in each concrete lift will be properly treated.
- Planned horizontal joint between two concrete lifts will be properly treated.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA revisions have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 056 Dated July 9, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

NRC RAI Number: 02.05.04-37

In response to RAI 02.05.04-8, you stated that more detailed excavation and backfill plans are currently being developed, and you concluded that common fill at the site, based on the detailed plans, will not have adverse impact on the foundation stability. In order for us to better evaluate the foundation stability, please provide updated excavation and backfill plans.

VCSNS RESPONSE:

Revised FSAR Figure 2.5.4-219 (Sheets 1 & 2) show the general extent of the planned excavations for the Unit 2 and 3 power blocks and the location of profiles through the excavated and backfilled areas. The outline of the approximate extent of excavation shown in each figure is that of the temporary H-pile (soldier pile and lagging) retaining walls which will provide adequate clearance for construction of the power block structures and components.

Revised FSAR Figures 2.5.4-220 through 2.5.4-223 show cross-sections of structure foundations A-A through D-D, respectively. The cross-sections show the planned extent of the concrete fill, structural fill, and common fill, and the temporary H-pile wall. As shown in the cross-sections, the structural fill beneath the Turbine, Annex, and Radwaste buildings extends down to rock and approximately 40 - 50 feet horizontally beyond the foundation footprint of these buildings. The maximum vertical distance to rock is less than 60 feet below any major power block building foundation.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

Revise the 2nd paragraph of FSAR Subsection 2.5.4.5.1 to read as follows:

> To obtain plant grade of about El. 400 feet, the natural ground surface is leveled by excavating up to 28 feet of residuum and saprolite. The remainder of the residuum, saprolite and partially weathered rock beneath the power block is excavated down to top of ~~sound~~ rock using temporary ~~slopes~~ retaining walls (soldier pile and lagging with tiebacks) for support of the near-vertical excavations (as depicted ~~shown~~ on Figures 2.5.4-219 ~~220~~ through 2.5.4-223). Temporary construction slopes are used in some limited areas beyond the retaining wall where the excavation is deeper than about 40 – 50 feet and for the access ramps and circulating water pipe trenches.

The natural soil at the two units is excavated to the top of sound rock which varies from as high as El. 384 feet to as deep as about El. 312 feet. The temporary construction slopes are (typical) 2-horizontal to 1-vertical (2H:1V), benched about every 20 feet.

Revise FSAR Subsection 2.5.4.5.2.1 to read as follows:

Excavation in the soils (Layers I and II) and any existing fills is achieved with conventional excavating equipment. Excavations will adhere to OSHA regulations (Reference 236) when less than 20 feet high. As noted in ~~the previous subsection~~ Subsection 2.5.4.5.1, ~~a cut with benched (typical) 2-horizontal to 1-vertical (2H:1V) slopes is used to support the power block excavation. The slopes have benches at about every 20 feet of height.~~ temporary tied-back retaining walls are used to support the near-vertical excavations. Temporary construction slopes are used in some limited areas beyond the retaining wall where the excavation is deeper than about 40 to 50 feet and for the access ramps and circulating water pipe trenches. Since the saprolitic soils can be highly erosive, even temporary slopes cut into the saprolite are sealed and protected.

Revise the 1st paragraph of FSAR Subsection 2.5.4.5.2.2 to read as follows:

Excavation in Layer III (PWR) rock is achieved using conventional earthmoving equipment. ~~A benched (typical) 2-horizontal to 1-vertical (2H:1V) slope is used to support the excavation.~~ Temporary retaining walls are used to support the near-vertical excavations.

Revise FSAR Figures 2.5.4-219 through 2.5.4-223 as shown below:

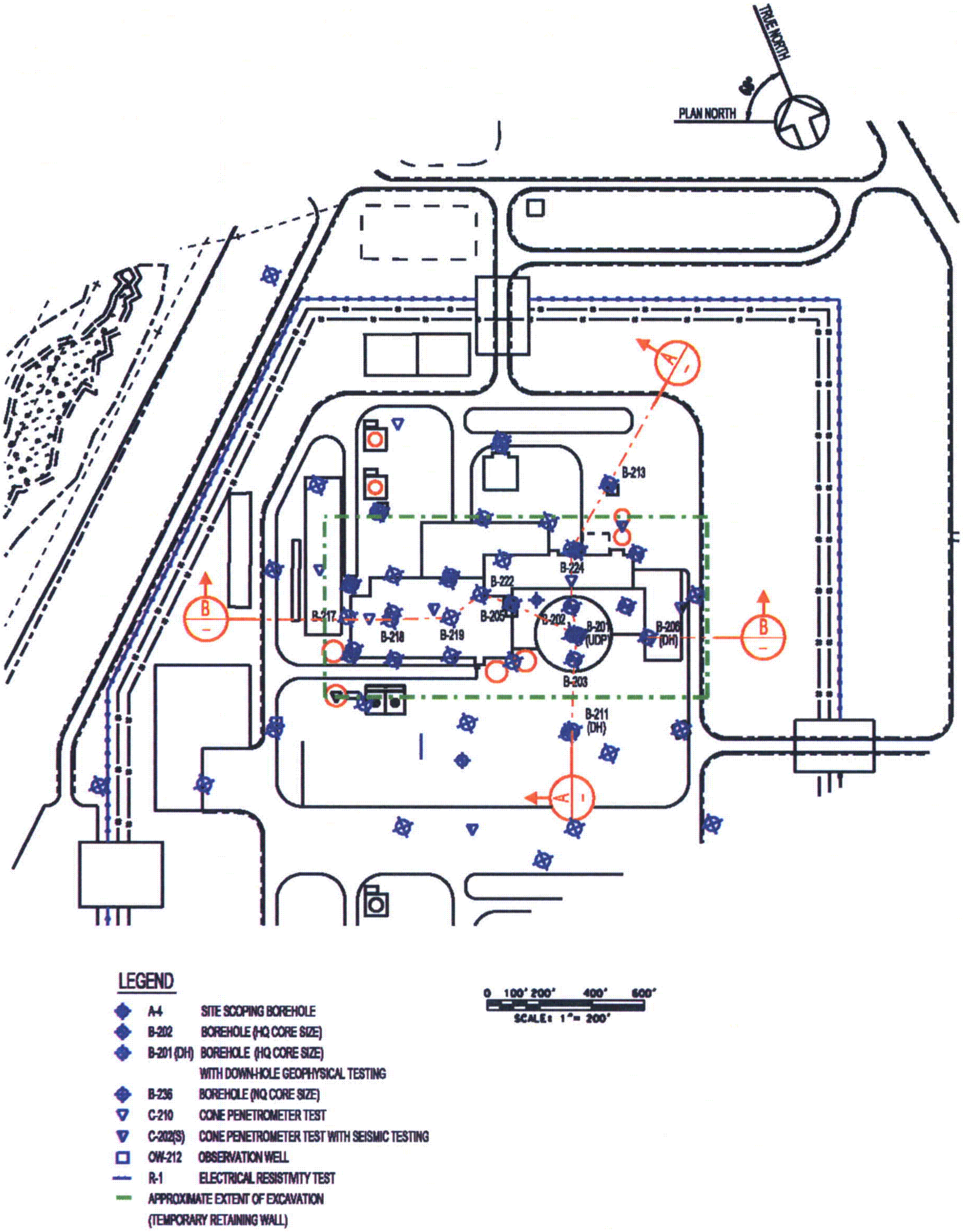

Figure 2.5.4-219: Profile Location Map Showing Excavation Geometry, Unit 2 (Sheet 1 of 2)

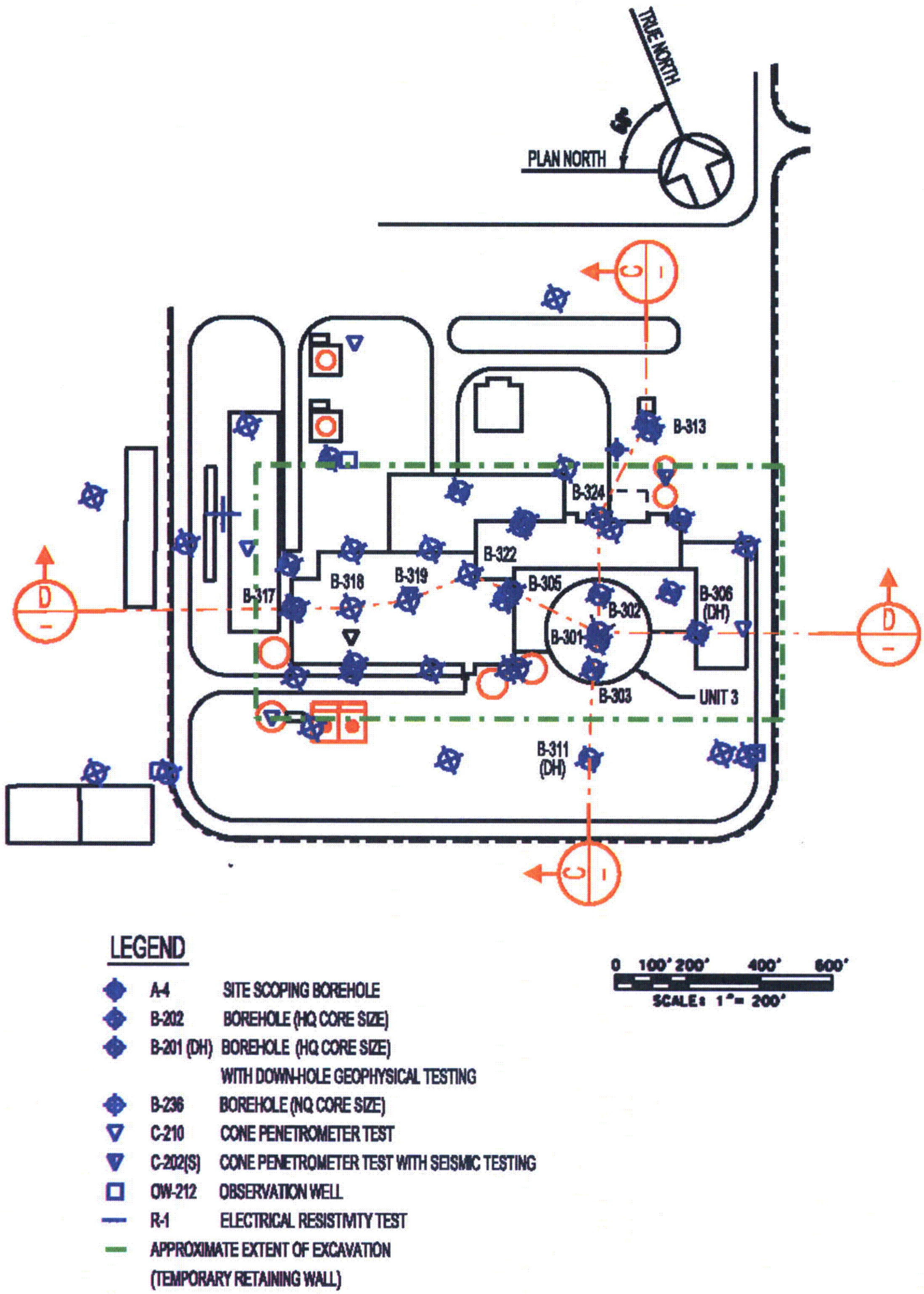

Figure 2.5.4-219: Profile Location Map Showing Excavation Geometry, Unit 3 (Sheet 2 of 2)

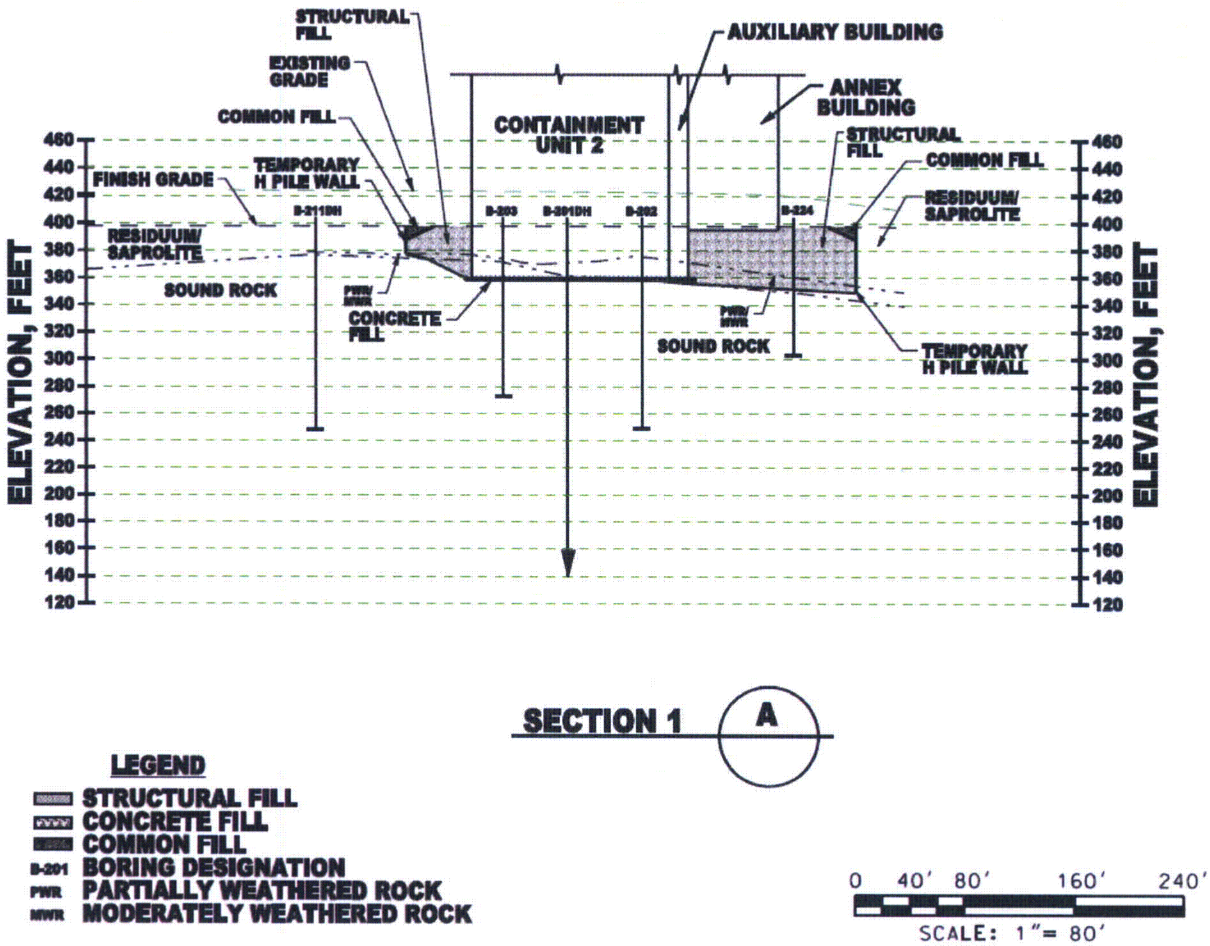

Figure 2.5.4-220: Cross-Section of Structure Foundation A-A

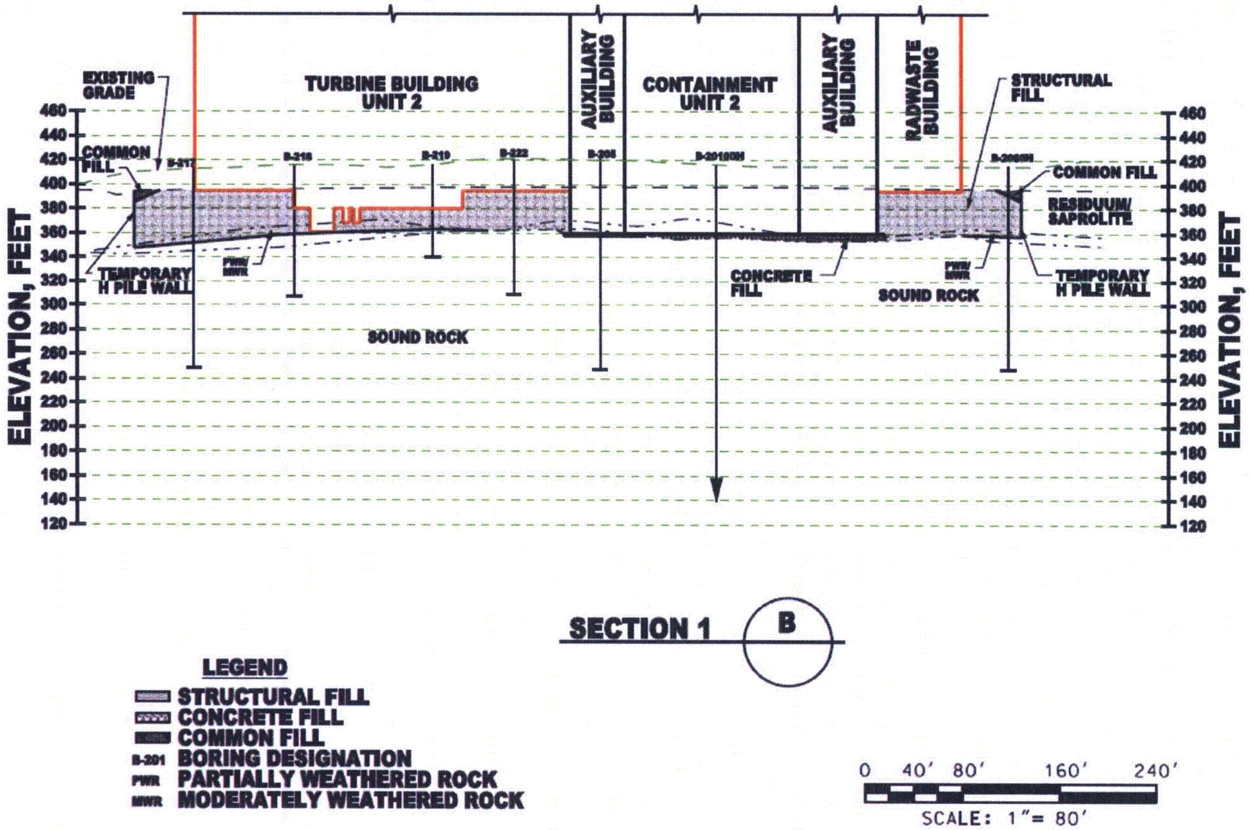

Figure 2.5.4-221: Cross-Section of Structure Foundation B-B

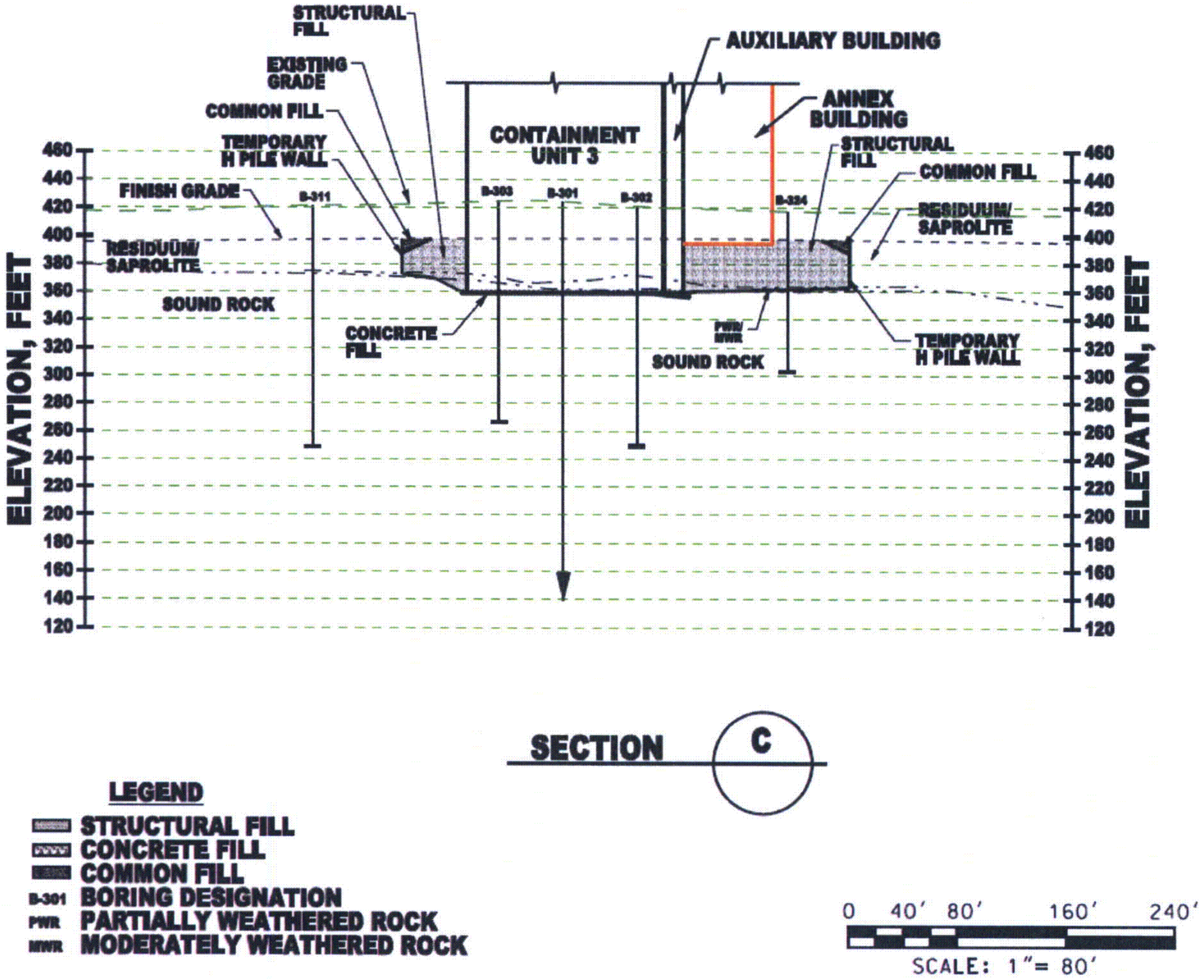

Figure 2.5.4-222: Cross-Section of Structure Foundation C-C

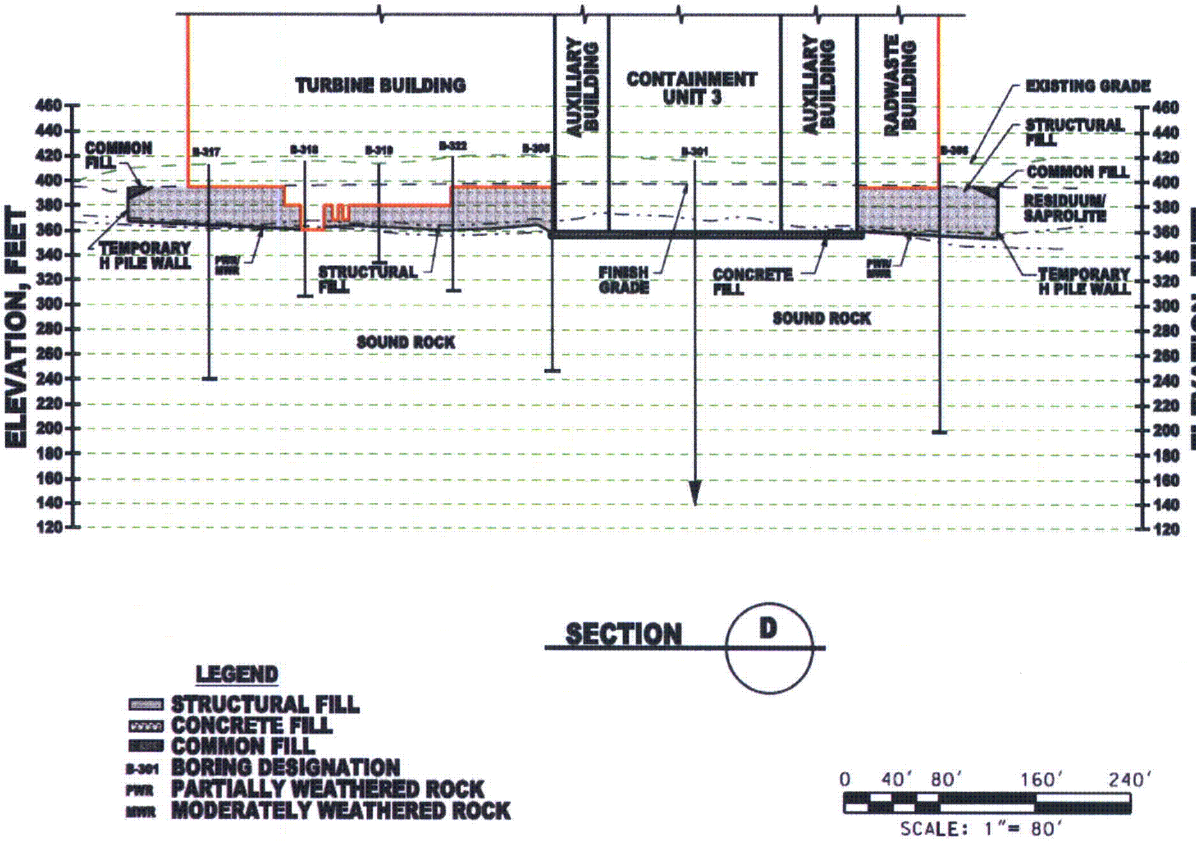

Figure 2.5.4-223: Cross-Section of Structure Foundation D-D

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 056 Dated July 9, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

NRC RAI Number: 02.05.04-38

In RAI 02.05.04-9, we asked for justification for the selected peak ground acceleration values and factors of safety used in the liquefaction analysis for saprolite soils. But in your response to this RAI, you did not address this issue. Please provide additional information so that we can complete our site liquefaction potential evaluation.

VCSNS RESPONSE:

As discussed in FSAR Subsection 2.5.4.8.3, the peak ground acceleration for the two units is shown in FSAR Figure 2.5.4-242 and shows 0.55g for the Unit 2 profile and 0.42g for the Unit 3 profile. As noted in FSAR Subsection 2.5.4.8.3, these peak ground accelerations are from PSHAKE analyses described in FSAR Subsection 2.5.4.7.3, including the use of 60 randomized soil and rock profiles. The 0.55g value was used for the liquefaction analysis of both units.

FSAR Subsection 2.5.4.8.4 indicates that Regulatory Guide 1.198 considers factors of safety (FS) values between 1.1 and 1.4 to be moderate. As noted in the FSAR, the FS value of 1.25 used in the liquefaction analysis of the saprolitic soils is considered adequate. This is the average of the "moderate" values, and can be considered somewhat conservative as well as adequate for these soils since they will be completely removed from below and around all major structures including the Nuclear Island, and liquefaction of these soils will not affect the safety-related structures of the plant.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response

ASSOCIATED ATTACHMENTS:

None